California High-Speed Train Project

Request for Proposal for Design-Build Services

RFP No.: HSR 11-16
Design Criteria

<u>**Note:**</u>
For additional changes to the Design Criteria, see the *Addenda Change Log for Addendum No. 3.*

Appendix 10.A – Guidelines for Geotechnical Investigations

1.0 Purpose

These guidelines represent a preferred, but not necessarily the only required actions needed for the development of additional geotechnical investigations. These guidelines convey a minimum standard of care in performing geotechnical investigations. These are not intended as prescribed site investigation criteria or checklists.

2.0 Geotechnical Investigation Guidelines and requirements

Geotechnical investigations are to be performed by a geotechnical engineer in collaboration with an engineering geologist, both of which are licensed in the State of California. The level of geotechnical investigation performed shall consider the engineering needs and amount of information necessary to achieve performance criteria, complete the design, and mitigate construction risks. Guidelines for advancing the geotechnical investigations are described in the following sections.

The geotechnical engineer/engineering geologist will be required to present the investigation results in a Geotechnical Data Report (GDR) document that contains the factual information/ data gathered during the geotechnical investigations. The GDR shall minimally contain the following information:

- Summarize and reference to separate geologic hazards report
- Description and discussion of the site exploration program
- Logs of all borings, trenches, and other site investigations
- Description and discussion of field and laboratory test programs
- Results of field and laboratory testing

The high cost component of geotechnical investigations is borehole drilling; therefore, planning of the geotechnical investigations shall maximize the use of existing geologic and subsurface data, and optimize the use of geophysical testing and Cone Penetration Tests (CPTs) where warranted in order to minimize the amount and cost of drilling required and still achieve a level of knowledge commensurate with good engineering practice and judicious judgment for similar locations and applications. Geotechnical investigations shall not begin until certain project specific information is gathered as set forth in the following sections.

2.1 Standards and Key Geotechnical Investigation Reference Documents

The ASTM test methods and FHWA manuals are considered the most comprehensive and applicable guideline documents for geotechnical investigation of the CHSTP as well as federal transportation projects. Chapter 6 of the 2008 FHWA Project Development and Design Manual (PDDM) provides an overview of practice for geotechnical work and direction for understanding policies and standards for geotechnical work performed by the FLH. The PDDM also provides a portal to technical information and presents a high-level source of technical guidance with regard to what needs to be accomplished. The corresponding 2007 FHWA Geotechnical Technical Guidance Manual (GTGM) provides guidance as to how the work shall be done. The GTGM also provides guidance for activities where standards and standard practices do not exist and provides access to and guidance for the use of new technologies. For soil and rock logging, classifications, and presentation, refer to 2010 Caltrans Soil and Rock Classification, Classification, and Presentation Manual.

2.2 Geotechnical Investigation Goals

The goals of geotechnical investigations project are to:

1. Perform additional subsurface investigations to supplement existing geotechnical data for design of structural elements including bridges, retaining walls, at-grade structures, cut-and-cover tunnels, large culverts, signs, and signals along the proposed alignment.

2. Identify the distribution of soil and rock types within the project limits and assess how the material properties will affect the final design and construction of the project elements.

3. Define the groundwater and surface water regimes, especially, the depth, and seasonal and spatial variability of groundwater or surface water within the project limits. The locations of confined water-bearing zones, artesian pressures, and seasonal or tidal variations shall also be identified.

4. Identify and characterize any geologic hazards that may be present within or adjacent to the project limits (e.g., faults, landslides, rockfall, debris flows, liquefaction, soft ground or otherwise unstable soils, seismic hazards). These items are vital pieces of the overall geotechnical exploration process, and the investigators must ensure that these elements are addressed.

5. Assess surface hydrological features (infiltration or detention facilities) that are required, as well as determine pond slope angle and infiltration rates to enable estimation of the size and number of those facilities required for the project.

6. Identify suitability of onsite materials as fill and/or the suitability of nearby materials sources.

7. For structures including bridges and cut-and-cover tunnels, large culverts, signs, signals, walls, or similar structures, provide adequate subsurface information for final design and construction.
8. For tunnels, trenchless technology, or ground improvement, provide adequate information to determine the viability of construction methods and potential impacts to adjacent facilities.
9. For landslides, rockfall areas, and debris flows, provide adequate information to determine stabilization or containment methods for design and construction.
10. Develop design soil properties for engineering evaluations, including dynamic analysis.
11. Perform chemical assessment of groundwater and soil for the impact evaluation of existing soil and groundwater on foundation materials.

2.3 Sequence of Geotechnical Investigations

Details on performing geotechnical investigations are provided in Section 2.4 and shall follow the general sequence listed below.

1. Review the scope of project requirements to obtain a clear understanding of project goals, objectives, constraints, values, and criteria. This information may consist of:
   - Project location, size and features
   - Project element type (bridge, tunnel, station, embankment, retaining wall, etc.)
   - Project criteria (alignments, potential structure locations, approximate structure loads, probable bridge span lengths and pier locations, and cut and fill area locations)
   - Project constraints (context-sensitive design issues, right-of-way, environmental and biological assessments and permitting)
   - Project design and construction schedules and budgets
2. Review of available geologic and geotechnical data.
3. Initiate and prepare geotechnical investigations. Identify the anticipated required analyses and key engineering input for the analyses.
4. Perform field reconnaissance and geological mapping. Obtain right-of entry where required.
5. Finalize the Geotechnical Investigation Plan (GIP) and submit to the Authority. Obtain permits and rights-of-entry.
6. Perform exploration and laboratory testing for final design.

7. Compile and summarize data for use in performing engineering analyses, and prepare geotechnical data reports.

2.4 Planning Geotechnical Investigations

The planning process for geotechnical investigations requires evaluating the appropriate number, depth, spacing, and type of exploration holes, as well as sampling intervals and testing frequencies. The involvement of engineering geologists (supporting the geotechnical engineer) is critical throughout the investigation process, from initial exploration planning through the characterization of site conditions, to assure consistency for geologic interpretation of subsurface conditions in support of developing parameters for use in phased engineering design and construction.

The geotechnical investigation program shall be carried out in phases.

2.4.1 Desk Study

Review of subsurface conditions based on existing geological and subsurface data.

All relevant available information on the project site shall be reviewed. Available data may consist of reports, maps, journal articles, aerial photographs, historical records of previous investigations by agencies, as-built plans from construction of existing facilities, and communication with individuals with local knowledge. A Geologic Hazards Report shall be prepared by a California Certified Engineering Geologist in advance of geotechnical investigations. The report shall be reviewed and utilized as a basis for geologic characterization and potential geologic hazards, and for siting of proposed subsurface exploration points. The results of the geologic and seismic hazard evaluation shall be collaborated with the project geotechnical engineer. Other sources of available information include the California Geological Survey (CGS), the United States Geological Survey (USGS), Caltrans archived Logs of Test Borings (LOTBs), the GIS database developed as part of the CHSTP, and data in individual city and county records and archives.

2.4.2 Field Reconnaissance

Field reconnaissance shall be conducted jointly by the geotechnical engineer and the Certified Engineering Geologist after the desk study is completed. The following factors shall be evaluated by the field reconnaissance:

- Geologic Report Reviews – The geotechnical engineer and engineering geologist responsible for the geotechnical investigations shall review and become familiar with geologic site characterizations and any identified geologic hazards provided in geologic hazards evaluation reports.

- Environmental Considerations – Potential impacts the project may have on subsurface materials, landforms, and the surrounding area shall be identified, and assessed to determine if project areas are governed by special regulations or have protected status.

- Explorations – The type(s) and amount of exploration and the kinds of samples that would best accomplish the phased project needs shall be evaluated.

- Drilling Logistics – The type, approximate locations, and depths of geotechnical explorations shall be defined, and approximate routes of access to each exploration location shall be determined. Make note of any feature that may affect the geotechnical investigation program, such as accessibility, structures, overhead utilities, evidence of buried utilities, or property restrictions. Evaluate potential water sources for use during borehole drilling operations. Evaluate potential concerns that may need to be addressed while planning an exploration program (permits, buried or overhead utilities clearance, equipment security, private property, etc.).

- Permits – The various types of permits that may be required shall be assessed, and all applicable jurisdictions shall be considered, which could include partner agencies, adjoining properties including railroads, Caltrans, regulatory agencies, and state and local government agencies. Local government agencies requirements could include regulations, codes, and ordinances from city, county, and departments of public works having jurisdiction. Permits could include right-of-entry, drilling and well permits, special use permits, lane closure and traffic control plans, utility clearances, etc.

2.4.3 General Subsurface Profiles

The general subsurface profiles, once developed, will present an overall geologic conditions of the areas under study and allow the geotechnical engineer (in collaboration with the engineering geologist) to determine the locations of supplementary explorations for final design and construction.

2.4.4 Carry Out Geotechnical Investigations In Stages

For areas where there are no existing subsurface investigation data, conduct geophysical testing such as Spectral Analysis of Surface Wave (SASW), Multi-channel Analysis of Surface Wave, (MASW), Suspension PS Logging, Cross-hole Seismic Logging, seismic refraction tests, seismic reflection tests, or a combination of the above to measure shear wave and P-wave velocities in situ and to generalize the subsurface conditions prior to drilling CPTs and borings. The sequence of site investigation shall be as follows:

- Geophysical testing – To determine the general subsurface conditions for areas with no available existing geologic data.

- CPTs – To confirm the general subsurface conditions with measurements of pore water pressure and shear wave velocities with depth by means of using a combination of seismic cones, CPTu, and CPTs.
- Borings – To refine the general subsurface conditions after CPTs are performed. Install observation wells or piezometers and inclinometers where necessary to confirm groundwater table levels and ground movement in the field. Perform suspension PS logging or cross-hole seismic logging at deep boreholes (180 feet or deeper) in structures located over river crossings or unusual geologic conditions [1], and other boring locations selected by the geotechnical engineer in collaboration with the engineering geologist.

[1] Unusual Geologic Conditions **–** Structures that are subject to unusual geologic conditions, including geologic hazards outlined in the *Geotechnical* chapter of the Design Criteria. This includes structures founded upon:

- Soft, collapsible, or expansive soil
- High groundwater table (within 5 feet below ground surface)
- Soil having moderate to high liquefaction and other seismically induced ground deformation potential
- Soil of significantly varying type over the length of the structure
- Fault Zones
- Unusual geologic conditions shall be defined within the Geotechnical Reports.

2.5 Surface Explorations

Standards for surface exploration methods are provided in PDDM Section 6.3.2.2, and technical guidance is provided in GTGM Section 3.2.2. Geologic field mapping of surficial soil and rock units and measurements of rock discontinuities shall begin by observing, measuring, and recording of exposed rock structure data at existing road cuts, drainage courses, and bank exposures, as well as portal locations where profiles transition from underground segments to elevated structures or at-grade reaches. Where rock exposures exist, mapping shall include initial characterization of rock mass rating, weathering, texture, overall quality, and discontinuity characteristics.

The objective of these observation and data collection efforts is to confirm the general types of soil and rock present, and topographic and slope features. For rock slopes, performance of slopes and the rockfall history are important indicators of how a new slope in the same material will perform. In addition to plotting data on a site plan or large-scale topographic map, preparation of field-developed cross sections is a valuable field method.

2.6 Subsurface Explorations

Relative advantages (economy, data quality, data collection time) of various methods of subsurface investigation should be considered in selecting the exploration plan. For example, geophysical methods and CPTs, which are relatively cheap and faster in operations, shall be conducted first, then followed by conventional test borings in specific situations.

Standards for performing subsurface explorations are provided in PDDM Section 6.3.2.2, and technical guidance is provided in GTGM Section 3.2.2. A guideline for the type of equipment and frequency of use for various types of investigations is presented in GTGM Exhibit 3.2-E. Additional guidance is contained in Caltrans (2007) logging manual.

The scope of the investigations shall reflect the anticipated subsurface and surface conditions and the preliminary results presented in the GDR during the bidding phase. Some factors that may impact the prioritization (sequence order ranking), method, number, and depth of subsurface explorations include the potential geologic hazards identified and geology (soil and rock units), landslides, slope stability, rockfall, rip-ability, fill suitability, expansive soils, compressible or collapsible soils, groundwater and hydrogeology, ground-borne vibration and noise transmissivity, erosion, temporary shoring, and excavation slopes. The level of investigation, priority, and scope of work for each component shall be developed in accordance with the geotechnical investigation guidelines set forth in these guidelines.

- Geophysical Methods – Spectral Analysis of Shear Wave (SASW) or Multi-channel Analysis of Shear Wave (MASW) in conjunction with suspension logging and/or cross-hole seismic logging shall be conducted to determine in situ shear wave and primary (P) wave velocities with depth. Shear wave and P-wave velocities are the key dynamic properties for seismic design and shall be measured in situ during geotechnical investigations.

  Standards for geophysical methods are provided in PDDM Section 6.3.2.3.2. The primary source supporting the guidance is FHWA DTFH68-02-P-00083 Geophysical Methods Technical Manual (2003). Secondary sources are NHI 132031 and USACE EM 1110-1-1802. Generally, geophysical methods are used as a reconnaissance investigation to cover large areas and/or to supplement information between boreholes. These exploration techniques are most useful for extending the interpretation of subsurface conditions beyond what is determined from small-diameter borings. The methods presented in FHWA (2003) shown as Exhibit 3.2-F of the GTGM are some of the most common. The reliability of geophysical results can be limited by several factors, including the presence of groundwater, non-homogeneity of soil stratum thickness, gradation or density, and the range of wave velocities within a particular stratum. Subsurface strata that have similar physical properties can be difficult to distinguish with geophysical methods. Geophysical methods are also applicable for testing ground-borne vibration transfer mobility of subsurface conditions, and assessment of this parameter is considered important for HST systems. The reference document for this testing is titled, "High-Speed Ground Transportation Noise and Vibration Impact Assessment," FRA Report No. 293630-1, December 1998.

- Cone Penetration Test, Seismic Cones, and Piezocone Penetrometer Test – CPT is a specialized quasi-static penetration test where a cone on the end of a series of rods is pushed into the ground at a constant rate and continuous or intermittent measurements are made of the resistance to penetration of the cone. This test can be used in sands or clays, fibrous peat or muck that are sensitive to sampling techniques, but not in rock, dense to very dense sands, or soils containing appreciable amounts of gravel, and cobble. The CPT is relatively inexpensive in comparison to borings, but it can only be used to supplement sampled borings because no samples are obtained so that no positive identification of soil types can be made out of the CPTs.

Piezocones are electric penetrometers that are capable of measuring pore-water pressures during penetration. When equipped with time-domain sensors, cones can also be used to measure shear wave velocity.

Tests are conducted in accordance with ASTM D 3441 (mechanical cones) and ASTM D 5778 (piezocones). References: TRB-NCHRP synthesis report 368 (2007), and FHWA-SA-91-043.

Many correlations relating CPT data to soil types and engineering properties have been published. These correlations can be used for design of spread footings and piles.

- Test Borings – Guidance for selection of the applicable exploration methods is presented in PDDM Exhibit 6.3-A (borings). Methods for exploratory borings shall be in accordance with AASHTO and ASTM standards. Detailed information on drilling and sampling methods is given in NHI132031 which lists applicable American Association of State Highway and Transportation Officials (AASHTO) and ASTM drilling and sampling specifications and test methods. Additional references include AASHTO MSI-1, FHWA GEC-5, FHWA-ED-88-053, National Highway Institute (NHI) 132012, NHI132035, USACE EM 1110-1-1804, USACE EM 1110-1-1906, FHWA-FL-91-002, and Caltrans (2007).

For the rotary wash drilling method, the drilling fluid in boreholes shall be kept above the groundwater level at all times. Rapid fluctuations in the level of drilling fluids shall be avoided. The boreholes shall be thoroughly cleaned prior to taking samples. Drill cuttings shall be collected and disposed of in accordance with applicable regulations.

Disturbed samples can be used for determining the general lithology of soil deposits, for identifying soil components and general classification purposes, and for determining grain size, Atterberg limits, and compaction characteristics of soils. The most commonly used in-situ test for surface investigations is the Standard Penetration Test (SPT), AASHTO T 206. The use of automatic hammers for SPT is highly recommended, and standard drop height and hammer weight must be maintained. The SPT values obtained with non-automatic hammers are discouraged and are allowed when calibrated by field comparisons with standard drop hammer methods. The SPT dynamic analyzer shall be used to calibrate energy of the SPT equipment at the site at least at the start of the project and bi-weekly for long-duration site investigations. More frequent use of the SPT dynamic analyzer is encouraged. For automatic hammers, calibrate the system to provide approximately 60% energy so that an energy correction is not required and N60 values will be obtained directly.

Undisturbed samples shall be obtained in fine-grained soil strata for use in laboratory testing to determine the engineering properties of those soils. Specimens obtained by undisturbed sampling methods may be used to develop the strength, stratification, permeability, density, consolidation, dynamic properties, and other engineering characteristics of soils. Disturbed and undisturbed samples can be obtained with a number of different sampling devices, as summarized in Table 7 of FHWA GEC-5 and Table 3-4 of NHI 132031.

It will be the responsibility of the geotechnical engineer to obtain enough testable samples of rock and soil to complete the agreed-upon laboratory testing program. The quantity of each type of test conducted shall be proposed by the geotechnical investigation consultant to adequately characterize each soil or rock unit encountered. Therefore, adequate subsurface exploration and sampling will be necessary to obtain sufficient sample quantity for subsurface characterization.

- Sandy or Gravely Soils Sampling – The SPT (split-spoon) samples shall be taken at 5-foot intervals or at significant changes in soil strata. Continuous SPT samples with a gap of at least 6 inch between two consecutive tests are recommended in the top 15 feet of borings made at locations where spread footings may be placed in natural soils. SPT bagged samples shall be sent to lab for classification testing and verification of field visual soil identification. Modified California (MC) and/or California (C) samplers shall not be used in these soils.

- Silt or Clay Soils Sampling – The SPT and undisturbed thin wall tube samples shall be taken at 5-foot intervals or at significant changes in strata. Take alternate SPT and tube samples in same boring or take tube samples in separate undisturbed boring. Tube samples shall be sent to lab to allow consolidation testing (for settlement analysis) and strength testing (for slope stability and foundation-bearing capacity analysis). The tube samples shall be retrieved by pushing soil out in the same direction that it entered the tube (i.e., through the top of the tube sampler; do not reverse and push it back out of the bottom). Field vane shear testing is also recommended to obtain in-place shear strength of soft clays, silts, and rotted peat.

- Rock Sampling – Continuous cores shall be obtained in rock or shales using double- or triple-tube core barrels. In structural foundation investigations, core a minimum of 10 feet into rock to ensure it is bedrock and not a boulder. Core samples shall be sent to the lab for possible strength testing (unconfined compression) if for foundation investigation. Percent core recovery and rock quality designation (RQD) value shall be determined in field or lab for each core run and recorded on the boring log. Additional guidelines for rock coring are described later in this section and in the reference manuals.

- Groundwater in Borings – Water level encountered during drilling, at completion of boring, and at 24 hours after completion of boring shall be recorded on the boring log. In low permeability soils such as silts and clays, a false indication of the water level may be obtained when water is used for drilling fluid and adequate time is not permitted after boring completion for the water level to stabilize (more than one week may be required). In such soils, a plastic pipe water observation well shall be installed to allow monitoring of the water level over a period of time. Seasonal fluctuations of water table shall be determined where fluctuation will have significant impact on design or construction (e.g., borrow source, footing excavation, excavation at toe of landslide). Artesian pressure and seepage zones, if encountered, shall also be noted on the boring log. In landslide investigations, slope inclinometer casings can also serve as water observation wells by using leaky couplings (either normal aluminum couplings or PVC couplings with small holes drilled through them) and pea gravel backfill. The top 1 foot or so of the annular space between water observation well pipes and borehole wall shall be backfilled with grout, bentonite, or sand-cement mixture to prevent surface water inflow, which can cause erroneous groundwater level readings.

- Probes, Test Pits, Trenches, and Shafts – Guidance for selection of the applicable exploration methods is presented in PDDM Exhibit 6.3-B (probes, test pits, trenches, and shafts), and GTGM Section 3.2.3.5. The recommended primary reference is NHI 132031. Additional guidance is contained in AASHTO MSI-1 and Caltrans 2007. Exploration pits and trenches performed by hand, backhoe, or dozer allow detailed examination of the soil and rock conditions at shallow depths and relatively low cost. Exploration pits can be an important part of geotechnical explorations where significant variations in soil conditions occur (vertically and horizontally), large soil and/or non-soil materials exist (boulders, cobbles, debris) that cannot be sampled with conventional methods, or buried features must be identified and/or measured. Upon completion, the excavated test pit shall be backfilled and compacted with the excavated material or other suitable soil material, and the surface shall be restored to its previous or approved condition.

- Soil Resistivity Testing – The ability of soils to conduct electricity can have a significant impact on the corrosion of buried structures and the design of grounding systems. Accordingly, subsurface investigations shall include conducting appropriate investigations to obtain soil resistivity values. The following information and methodologies are recommended.

  – Soil resistivity readings shall be obtained to determine the electric conduction potential of soils at each traction power facility (supply/paralleling/switching station), which are spaced at approximately 5-mile intervals.

  – Resistivity measurements shall be obtained in accordance with Institute of Electrical and Electronics Engineers (IEEE) Standard 81-1983 - IEEE Guide for Measuring Earth Resistivity using the four-point method for determining soil resistivity. IEEE states that the four-point method is more accurate than the two-point method.

- Standards for Boring Layout and Depth – Standards for boring layout and depth with respect to structure types, locations and sizes, and proposed earthwork are provided in these guidelines.
- Standards for Sampling and Testing From Borings – Minimum standards for disturbed and undisturbed soil and rock are presented in Exhibit 6.3-D of PDDM, and Section 3.2.3.3 of GTGM.
- Rock Coring – Standards for soil and rock classification are provided in PDDM Section 6.3.2.3.4, and technical guidance is provided in GTGM Section 3.2.3.4. The International Society of Rock Mechanics (ISRM) classification system shall be followed for rock and rock mass descriptions, as presented in FHWA GEC-5 FHWA-IF-02-034. The primary source supporting the standards and guidance is NHI 132031, and a secondary source is AASHTO MSI-1. Because single-tube core barrels generally provide poor recovery rates, the double- or triple-tube core barrel systems shall be used. To protect the integrity of the core from damage (minimize extraneous core breaks), a hydraulic ram shall be used to expel the core from the core barrel. Rock cores shall be photographed in color as soon as possible after being taken from the bore hole and before laboratory testing.

If rock is encountered in boreholes within the planned depth of drilling, continuous rock coring shall be performed in accordance with the following procedures. Rock coring shall be performed using a triple tube HQ coring system or a larger-diameter, triple-tube coring system. The HQ system produces cores 2.4 inches in diameter. The advantage of the triple tube system is that a split liner is used to contain the core, which results in relatively minimal disturbance to the core. Where weak rock zones are encountered, soil sampling techniques may be used instead of coring to recover samples that would be relatively undisturbed and suitable for testing. These techniques include the use of samplers such as the Pitcher or MC samplers. The potential difficulty with these samplers is that they can be easily damaged by hard, gravel-size particles that are often mixed with the softer, clay-like matrix of the weathered rock. These difficulties will need to be considered when planning the exploration program.

Rock core samples shall be placed in plastic core bags or double wrapped in plastic wrap and placed in wooden core boxes and transported to a storage facility at the end of each day. An adequate number of core boxes shall be maintained on site at all times during field exploration activities. The core shall be photographed, taking at least one photo for each core box, and close-ups taken of special features such as shear zones or other features of special interest. The core box label shall be clearly visible within the photo. An experienced geologist shall study the core and edit the borehole log based on their observations. Cores boxes shall be maintained throughout the design process and construction, with cores that have been removed for testing duly indicated in the appropriate locations in each box.

In some rock slope applications, it is important to understand the precise orientation of rock discontinuities for the design. Standards for using orienting-recovered rock core are presented in NHI 132031. In special cases, boreholes can be photographed/imaged to visually inspect the condition of the sidewalls, distinguish gross changes in lithology, and identify fracture zones, shear zones, and joint patterns by using specialized television cameras. Refer to AASHTO MSI-1, Section 6.1.2.

- Care and Retention of Samples – Standards for soil and rock retention are provided in the *Geotechnical* chapter of the Design Criteria - subsurface investigation and data analysis. and technical guidance is provided in GTGM Section 3.2.3.7.

2.7 Soil and Rock Classification

Standards for soil and rock classification are provided in PDDM Section 6.3.2.4, and technical guidance is provided in GTGM Section 3.2.4. Soils shall be classified in accordance with the ASTM Unified Soil Classification System (USCS). Rock and rock mass descriptions and classification shall follow the ISRM classification system presented in FHWA GEC-5. Material descriptions are based on the visual-manual method, and materials classifications are based on laboratory index tests (ASTM D 2487). Additional guidance is contained in Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2007).

2.8 Exploration Logs

Standards for preparing exploration field logs are provided in PDDM Section 6.3.2.5, and technical guidance is provided in GTGM Section 3.2.5.

- Field Logs – Field logging shall be performed by a geologist or engineer under the direct supervision of a California registered geotechnical engineer or certified engineering geologist. Logging shall be performed in accordance with ASTM D 5434. The location information (e.g., station, offset, elevation, and/or state plane coordinates) of all the explorations are to be recorded on the field logs. Exploration locations shall be located at the time of drilling by GPS with at least sub-10-foot accuracy. The explorations shall eventually be located by a licensed land surveyor. Required documentation for test pits shall include a scale drawing of the excavation, and photographs of the excavated faces and spoils pile. Drilling and sampling methods and in-situ measurement devices that were used shall also be documented. The field logs shall contain basic reference information at the top, including project name, purpose, specific location and elevation, exploration hole, number, date, drilling equipment, procedures, drilling fluid, etc. In addition to the logging descriptions of soil and rock encountered during exploration, the depth of each stratum contact, discontinuity, and lens shall be recorded. The reason for terminating an exploration hole and a list/description of instrumentation (if any) or groundwater monitoring well installed shall be written at the end (bottom) of each exploration log.

- Final Logs – Exploration logs shall be prepared with the gINT boring/test pit log software platform, using the formatted boring record template standardized by Caltrans (illustrated as Figures 5-12 and 5-13 in the Caltrans logging manual, 2007 version). An explanation key, known as the Boring Record Legend shall always accompany exploration logs whenever they are presented. The standardized legends to be used for CHSTP are illustrated as figures 5-14 through 5-16 of Caltrans (2007). The final edited log shall be based on the initial field log, visual classification, and the results of laboratory testing. The final log shall include factual descriptions of all materials, conditions, drilling remarks, results of field and lab tests, and any instrumentation. Where groundwater observation wells or piezometers are installed, several measurements are usually necessary within a one-week timeframe following drilling to verify that measured groundwater levels or pressures have achieved equilibrium. As a minimum, final boring logs shall contain the information shown in NHI132031. AASHTO MSI-1 provides additional guidance regarding documentation for boring logs.

2.9 In-Situ Testing

Standards for performing in-situ testing are provided in PDDM Section 6.3.2.6, and technical guidance is provided in GTGM Section 3.2.6. The primary reference is NHI1 32031. In-situ testing is very beneficial for projects where obtaining representative samples suitable for laboratory testing is difficult. Field in-situ borehole tests can be correlation tests, strength and deformation tests and permeability tests. Correlation tests primarily consist of SPTs performed in accordance with ASTM D 1596 and AASHTOT 206, and Dynamic CPTs are performed in accordance with ASTM D 3441.

- In-situ soil tests may consist of the following:
  - Cone Penetration Test (CPT) – Refer to Section 2.6 above.
  - Pressuremeter Test – This test measures state of stress in-situ and stress/strain properties of soils by inflating a probe placed at a desired depth in a borehole. Tests are completed in accordance with ASTM D 4719. Reference FHWA-IP-89-008.
  - Flat-Plate Dilatometer Test – This test uses pressure readings from an inserted plate at the base of a borehole to determine stratigraphy and obtain estimates of at-rest lateral stresses, elastic modulus, and shear strength of loose to medium dense sands (and to a lesser degree, silts and clays). Tests are completed in accordance with ASTM D 6635. Reference FHWA-SA-91-044. Care and judgment shall be undertaken for this test as it often provides information that is difficult to interpret or relate to parameters needed for engineering design.
  - Field Vane Shear Test (VST) – This test is used on very soft to medium stiff cohesive soil or organic deposits to measure the undrained shear strength, remolded strength of

the soil and soil sensitivity. Field vane shear test may provide more reliable estimate of peak and residual shear strength in cohesive soils, as disturbance from sampling and testing in laboratory is avoided. Tests are completed in accordance with ASTM D 2573 and AASHTO 223. VST is often regarded as a valuable test to estimate peak and residual shear strength in cohesive soils as disturbance from sampling and testing in the laboratory can be avoided.

- Hydrogeologic testing in-situ may consist of the following:
  - Permeability Tests – Several in-situ hydraulic conductivity tests exist, with the most commonly used methods being the pumping test and the slug test. The selection of the appropriate aquifer test method for determining hydraulic properties by well techniques is described in ASTM D 4043. In general, refer to NHI1 32031, BOR Geology Manual, and NAVFACDM-7.1.
  - Pumping Test – The pumping test requires not only a test well to pump from, but also one to four adjacent observation wells to monitor the changes in water levels as the pumping test is performed. Pumping tests are typically used in large-scale investigations to more accurately measure the permeability of an area for the design of dewatering systems. Refer to ASTM D 4050.
  - Slug Test – The slug test is quicker to perform and much less expensive, because observation wells are not required. It consists of affecting a rapid change in the water level within a well by quickly injecting or removing a known volume of water or solid object, known as a slug. The natural flow of groundwater out of or into the well is then observed until equilibrium in the water level is obtained. Refer to ASTM D 4044.
  - Packer Tests – These tests are performed in a borehole by placing packers above and below the soil/rock zone to be tested. One method is to remove water from the material being tested (Rising Water Level Method). Another method is to add water to the borehole (Falling Water Level Method and Constant Water Level Method). A third method utilizes water under pressure rather than gravity flow. The coefficient of permeability that is calculated provides a gross indication of the overall mass permeability. Refer to FHWA-TS-89-045 and NHI1 32031.
  - Open Borehole Seepage Tests – Methods include "Falling Water Level," "Rising Water Level," and "Constant Water Level" and are selected based on the relative permeability of the subsurface soils and groundwater conditions. Further detail is provided in Chapter 6 of NHI1 32031.
  - Infiltration Tests – Two types of infiltrometer systems are available: sprinkler type and flooding type. Sprinkler types attempt to simulate rainfall, while the flooding type is applicable for simulating runoff conditions. Applications for these tests include the design of subdrainage and dry well systems. The most common application is the falling

head test, performed by filling (flooding) a test pit hole and monitoring the rate at which the water level drops. Refer to ASTM D 4043.

Handling and disposal (or permitted discharge to storm sewer system) of water generated from hyrdrogeologic field testing shall be the responsibility of the geotechnical engineer conducting the investigation work.

If the geotechnical engineer intends to use field tests not covered in the current ASTM or referenced standards, the proposed test methods shall be submitted to the Authority prior to start of testing.

2.10 Laboratory Testing of Soil and Rock

Standards for performing laboratory testing are provided in PDDM Section 6.3.2.7 and technical guidance is provided in GTGM Section 3.2.7. Sufficient laboratory testing shall be performed to represent in-situ conditions. Exhibit 3.2-J of the GTGM provides a guideline for estimating laboratory test requirements for the different types of geotechnical analysis. Chapters 7 through 10 of NHI 132031, GEC-5, and Chapters 2 and 3 of NHI 132012 provide overviews of testing and correlations, as well as criteria to consider when planning the scope of testing programs. Additional references include AASHTO MSI-1, NHI 132012, NHI 132035, USACE EM 1110-2-1906, FHWA-FL-91-002; and Kulhawy and Mayne (1990). Exhibits 3.2-K (soil) and Exhibit 3.2-L (rock) of GTGM present a summary of the predominant laboratory tests. The proposed workplans for laboratory testing programs shall be submitted for review. Testing shall be done at a Caltrans approved facility.

If the geotechnical engineer proposes to use laboratory tests not covered in the current ASTM or referenced standards, the Geotechnical Designer shall submit a variance of test methods to the Authority for approval prior to commencement.

2.11 Instrumentation and Monitoring

Standards for installing and monitoring geotechnical instrumentation are provided in PDDM Section 6.3.2.8, and technical guidance is provided in GTGM Section 3.2.8. Instrumentation is used to augment standard investigation practices and visual observations where conditions would otherwise be difficult to evaluate or quantify due to location, magnitude, or rate of change. The quantity and locations of proposed geotechnical instrumentation shall be selected to suit the anticipated conditions consistent with project objectives and design requirements. The geotechnical exploration work plan shall include instrumentation work detailing locations, installation procedures, and methods to be used. The work plan shall be submitted to the Authority prior to commencement. Additional information about inclinometers and piezometers are presented in Cornforth (2005).

3.0 Project Features Requiring Geotechnical Investigations

3.1 General

The CHSTP will require geotechnical investigations of the various project features. The referenced standards and technical guidance documents shall be utilized, in addition to the primary and secondary references, where listed. Guidelines for the approximate number and depth of various exploration methods are included. In addition to the general guidelines, the scope of the investigation for the various project features shall also reflect the anticipated subsurface and surface conditions, as well as the design phase level (whether preliminary or final). Some factors that may impact the method, number, depth, and prioritization of subsurface explorations include type of soil or rock, landslides, slope stability, rockfall, rippability, fill suitability, expansive soils, compressible soils, groundwater and hydrogeology, ground-borne vibrations, erosion, engineering design needs, temporary shoring, and excavation slopes.

The scope of investigation work for each component shall be developed in accordance with the guidelines contained in this section. The quantity, locations, and depths of proposed geotechnical exploration shall be selected to suit the anticipated conditions consistent with phased project objectives and design requirements. The geotechnical exploration work plan shall include information detailing methods to be used and proposed schedule. The preliminary work plan shall be submitted to the Authority prior to commencement. If the geotechnical engineer proposes to use exploration methods or frequencies that differ from the guidelines set forth herein or are not covered in the current reference standards, the Geotechnical Designer shall submit a variance for the proposed alternate exploration plans to the Authority for review and approval prior to commencement.

The geophysical testing and CPTs provide advantages over conventional test borings under specific situations and should be considered first.

3.2 Rail Alignment and Earthwork

Standards for investigations for the at-grade rail alignment and earthwork are provided in PDDM Section 6.3.1.2.1, and technical guidance is provided in GTGM Section 3.1.2.1. Explorations are made along the proposed at-grade rail alignment for the purpose of defining the geotechnical properties of materials. This information is used to:

- Design cut and fill slopes
- Assess material suitability for embankment construction
- Define the limits of potential borrow materials

- Assess the suitability of foundation materials
- Evaluate settlement or slope stability problems
- Quantify the depths of topsoil and volumes of material to be removed
- Design remedial measures in areas of poor materials
- Aid the designer of the rail roadbed subgrade section
- Identify geologic hazards such as liquefaction and landslides

For cuts and fills, test borings shall be advanced every 200 feet (erratic conditions) to 400 feet (uniform conditions) along the project alignment where cuts or fills are anticipated. For large cuts or fills (e.g., 30 feet or more in height) an additional boring near the top of the proposed cut and toe of the proposed fill to evaluate cut/fill feasibility and overall stability may be necessary. Depths of the borings shall be at least three times the vertical height of the fill (or 40-foot minimum depth) and at least 15 feet below the base of the cut. If soft or poor soils are encountered, additional depth to competent material or 10 feet into rock will be needed to define the subsurface conditions.

3.3 Structures

Standards for structures and geotechnical hazards are provided in PDDM Section 6.3.1.2.3, and technical guidance is provided in GTGM Section 3.1.2.3 and Exhibit 3.1-B Guideline "Minimum Boring" Criteria. Structures and geotechnical hazards will primarily consist of the following:

- Bridges and aerial structures (viaducts)
- Stations
- Buildings
- Retaining walls
- Tunnels and portals
- Large culverts
- Mast-arm supports (OCS, signals, message signs)
- Landslides
- Faults

For bridges, one boring shall be drilled at the substructure unit under 100 feet in width and two borings per substructure unit over 100 feet in width, both drilled to a depth of 20 feet below pile/shaft tip elevation or two times maximum pile group dimension, whichever is greater or to a depth of a minimum of 10 feet into bedrock. In addition, at least one seismic cone and one suspension PS logging shall be conducted at each bridge to measure shear wave and P-wave velocities in situ, each to a depth of 100 feet or deeper. The number of the seismic cones and suspension logging shall increase if the bridge is of multiple long spans (greater than 350 feet) and/or if the bridge is located in erratic soil conditions with soft, compressible and loose saturated soils.

For buildings and stations, one boring shall generally be made at each corner and one in the center. This may be reduced for small buildings. For extremely large buildings and stations or highly variable site conditions, one boring shall be taken at each support location. Refer to building foundation manuals and CBC (codes) for additional guidance in planning geotechnical investigations. In addition, areas of influence of the building/station and/or of surrounding geologic or geotechnical issues shall be considered in defining the extent of explorations.

For retaining walls, the minimum site exploration will be one boring and one CPT alternating at 100 to 200 foot intervals, each drilled to a depth of 0.75 to 1.5 times wall height or to a competent stratum if potential deep stability or settlement is a problem. The boring and CPT can be interchangeable and located at the front of and some in back of the wall face.

Due to the extreme variability of conditions under which tunnels are constructed and the complexity of the projects, it is difficult to provide specific recommendations for tunnel investigation criteria. In general, boring footage is typically on the order of 1.5 to 3.0 linear feet of borehole per route foot of tunnel, and site exploration budgets are typically on the average of three percent of the estimated tunnel cost. Criteria shall be established for each project reach on an individual basis and be based on the complexity of the geology and the length and depth of the tunnel. FHWA-IF-05-023 and U. S. National Committee on Tunneling (USNCTT, 1995) shall be considered the primary references.

For culverts, a minimum of 1 boring per major culvert drilled to a competent stratum or to a depth of twice the culvert height, whichever is less.

Standard foundations for sign bridges, cantilever signs, cantilever signals, and strain pole standards are based on allowable lateral bearing pressure and angle of internal friction of the foundation soils. The determination of these values may be estimated by SPT and CPT. One boring shall be made at each designated location. Borings shall extend 50 feet into suitable soil or 5 feet into competent rock. Deeper borings may be required for posts with higher torsional loads or if large boulders are anticipated. Other criteria are the same as for bridges.

In addition to the above structures, any structure such as signage or other design features shall be addressed with regard to their potential influence and evaluated, as needed.

3.4 Landslides – Slope Instability

Standards for investigations for landslides are provided in PDDM Section 6.3.1.2.4, and technical guidance is provided in Section 3.1.2.4 and Exhibit 3.1-B of the GTGM. A minimum of three borings shall be advanced along a line perpendicular to centerline or planned slope face to establish geologic cross sections for stability analysis. The number of cross sections depends on the extent of the slope stability problem. For active slides, place at least one boring each above and below the sliding area. The borings shall be extended to an elevation below active or potential failure surfaces and into hard stratum, or to a depth for which failure is unlikely because of geometry of the cross section. If slope inclinometers are used to locate the depth of an active slide, they must extend to a depth below the base of the slide. Observation wells and/or piezometers at selected depths will also be required to determine the groundwater table in the soil/rock mass.

3.5 Faults

At locations where active faulting is suspected to be coincident with or within the area of CHSTP operations and facilities, a geologic reconnaissance will be required to ground-truth mapped fault traces. This reconnaissance shall be carried out by means of interpretations of aerial photos, LiDAR data, satellite imagery, and topographic information. The locations shall be reviewed in the field to assess the presence of geomorphic features associated with faulting such as escarpments, pressure ridges, sag ponds, seeps/springs, vegetation contrasts, or deflected drainages. All such features shall be documented on a geologic field map. If sufficient field data is available to document that the fault or fault zone is outside the footprints of the high speed train operations, no further fault evaluation is required. Otherwise, a site specific investigation including paleo-seismic trenching will be necessary.

If existing paleo-seismic trenching data is available, it may be reviewed and used as a basis for locating the fault and providing its rupture characteristics for final design; however, if either a known active fault or suspected active fault is located near or at the location of a project facility, exploratory trenching across the fault will be required to assess its rupture characteristics for input to final design.

3.6 Materials Sources

Standards for investigations for materials sources are provided in PDDM Section 6.3.1.2.2, and technical guidance is provided in Section 3.1.2.2 and Exhibit 3.1-B of the GTGM. Borings shall be spaced every 100 to 200 feet. The depth of exploration shall extend 5 feet beyond the base of the deposit, or to a depth required to provide the needed quantity of borrow material. These investigations shall evaluate the quality and quantity of materials available at existing and prospective sources within the vicinity of a project. These materials could include gravel base, crushed surfacing materials, pavement and concrete aggregates, riprap, wall backfill, borrow excavation, and select backfill materials. The evaluation may consider existing government-owned material sources, existing commercial material sources, expansion of existing sources, and development of new material sources.

3.7 Hydrological Features – Infiltration and Detention Facilities

For surface hydrological features (infiltration or detention facilities) that may be needed, at least one boring per site shall be obtained to assess feasibility and define groundwater conditions. Boring depths will depend on the nature of the subsurface conditions encountered and the depth of influence of the geotechnical feature. Borings shall extend at least 20 feet below the likely base elevation of the facility, or five times the maximum anticipated ponded water depth, whichever is greater. Observation wells and/or piezometers shall be installed and monitored for at least 1 year to assess yearly highs and lows for the groundwater.

3.8 Pavement

Pavements are not a significant component of the HST trackway alignment design but will be an extensive design element for station areas, access roads, grade separations, and surface road reconstruction. Standards for investigations for pavement subgrade are provided in PDDM Chapter 6, Section 6.3.1.2.5 and Chapter 11, and technical guidance is provided in GTGM Section 3.1.2.5. Other sources supporting investigation standards and guidance are NHI 132031, AASHTO MSI-1, and FHWA GEC-5.

4.0 References

1. American Association of State Highway and Transportation Officials (AASHTO)
   - Manual on Subsurface Investigations, MSI-1, 1988.
   - Standard Recommended Practice for Decommissioning Geotechnical Exploratory Boreholes, AASHTO R 22-97, standard Specifications, 2005.
   - Specification for Transportation Materials and Methods of Sampling and Testing, Part II: Tests, HM-28-M, 2008.
2. American Society of Civil Engineers (ASCE) reference titled “Geotechnical Baseline Reports for Construction – Suggested Guidelines”, ASCE 2007.
3. American Railway Engineering and Maintenance of Way Association (AREMA) – Manual for Railway Engineering, 2008 Edition.
4. ASTM, Annual Book of ASTM Standards, 2008 Edition.
5. Caltrans, Soil and Rock Logging, Classification, and Presentation Manual, June 2010.

6. Cornforth, D.H., Landslides in Practice: Investigations, Analysis, and Remedial/Preventive Options in Soils, Chapter 4, John Wiley & Sons 2005.
7. Federal Highway Administration (FHWA):
   - Geotechnical Technical Guidance Manual, May 2007.
   - Project Development and Design Manual (Draft) – Chapter 6 - Geotechnical, April 2011.
   - Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications, FHWA-ED-88-053, 1988, revised February 2003.
   - Road Tunnel Design Guidelines, FHWA-IF-05-023, 2004.
   - Geophysical Methods - Technical Manual (Application of Geophysical Methods to Highway Related Problems, cooperatively with Blackhawk Geosciences), DTFH68-02-P-00083, 2003.
   - Soils and Foundations Workshop, NHI Course No. 132012, Volumes I and II FHWA-NHI-06-088, and FHWA-NHI-06-089, 2006.
   - Subsurface Investigations – Geotechnical Site Characterization, NHI Course Manual No. 132031, FHWA-NHI-01-031, 2002.
   - Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5, FHWA-IF-02-034, 2002.
8. Federal Railroad Administration (FRA), High-Speed Ground Transportation Noise and Vibration Impact Assessment, FRA Report No. 293630-1, December 1998.
9. ISRM, Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses, 1981.
10. Kulhawy, F.H. and Mayne, P.W., Manual on Estimating Soil Properties for Foundation Design, EPRI Report EL-6800, 1990.
11. U. S. Army Corps of Engineers (USACE), Geotechnical Investigations, Engineering Manual, EM 1110-1-1804, Department of the Army, 2001.
12. U. S. Army Corps of Engineers (USACE), Soil Sampling, Engineering Manual, EM 1110-1-1906, Department of the Army, 1996.

Appendix 10.B – Guidelines for Geotechnical Earthquake Engineering

1.0 Purpose

These guidelines represent a preferred, but not necessarily the only required actions needed for a particular design feature associated with earthquake engineering. These guidelines convey a minimum standard of care in performing earthquake engineering design. These are not intended as a prescribed design criteria or checklist.

2.0 Seismic Design Criteria

Seismic design criteria for geotechnical earthquake engineering have been established in terms of two levels of project performance criteria: No Collapse Performance Level (NCL) and Operability Performance Level (OPL) as noted in the *Seismic* chapter of the Design Criteria.

Geotechnical seismic design shall be consistent with the philosophy for structural design for the two performance levels. The performance objective shall be achieved at a seismic risk level that is consistent with the seismic risk level required for that seismic event. Slope instability and other seismic hazards such as liquefaction, lateral spread, post-liquefaction pile downdrag, and seismic movement/settlement may require mitigation to ensure that acceptable performance is obtained during a design seismic event. The geotechnical designer shall evaluate the potential for differential movement/settlement between mitigated and non-mitigated soils. Additional measures may be required to limit differential movement/settlements to tolerable levels both for static and seismic conditions. The foundations shall be designed to address liquefaction, lateral spread, and other seismic effects to prevent collapse. All earth-retaining structures shall be evaluated and designed for seismic stability internally and externally. Cut slopes in soil and rock, fill slopes, and embankments, especially those which could have significant impact on high-speed train (HST) operation, shall be evaluated for instability due to design seismic events and associated geologic hazards.

2.1 Liquefaction Triggering and Consequences

Evaluation of soil liquefaction triggering potential shall be performed in two steps. The first step involves evaluating whether the soil meets the compositional criteria necessary for liquefaction. These compositional criteria are presented in the design criteria manual.

For soils meeting the compositional criteria, the next step is to evaluate whether the design level ground shaking is sufficient to trigger liquefaction given the soil's in-situ density. If it is assessed that liquefaction will be triggered, the engineering consequences of liquefaction shall be evaluated. In addition to triggering for liquefaction, the geotechnical engineer shall consider the allowable deformation and the long-term, post-construction performance requirements for earth and fill conditions.

For fine-grained soils (especially soils that are potentially sensitive) that do not meet the compositional criteria for liquefaction, cyclic softening resulting from seismic shaking shall be performed.

Considering the range of criteria currently available in the literature, geotechnical engineers shall consider performing cyclic triaxial or simple shear laboratory tests on undisturbed soil samples to assess cyclic response for critical cases.

For gravels, field investigation methods appropriate for soil layers containing gravels include the Becker Hammer Penetration Test (BPT), Large Sampler Penetration Test (LPT), and small interval SPT. Seed et al. (2003) discusses different methods for performing liquefaction analysis in coarse and gravelly soils.

2.2 Liquefaction Triggering Evaluations

Liquefaction-triggering evaluations shall be performed for sites that meet the two design criteria established in the design criteria manaul:

CPT and/or CPTu (with pore water pressure measurement) shall be used as the primary method of field investigation for liquefaction analysis where it can be advanced without premature refusal. SPT shall be used as the primary liquefaction evaluation method where borings are performed. LPT, shear wave velocity (Vs), or BPT shall be used in soils difficult to test using SPT and CPT methods, such as gravelly soils. In addition, small interval SPT (blow counts measured for every 1 inch) shall be used in gravelly soils. More rigorous, nonlinear, dynamic, effective stress computer models may be used for site conditions or situations that are not modeled well by the simplified methods.

2.2.1 Simplified Procedures

All three simplified methods by Youd et al. (2001), Seed et al. (2003), and Idriss and Boulanger (2008) shall be used for liquefaction-triggering analysis for each boring and/or CPT. Results in terms of FOS shall be reported. Results of these analyses shall be interpreted according to the following. If the FOS values between the three methods are within 20% of each other, an average FOS shall be reported for that particular boring and/or CPT. If the FOS values from these three methods vary by more than 20% and use of the more conservative results for design would have significant cost consequences, some additional evaluations may be warranted. The additional evaluations shall include an assessment of which method best applies to this specific case, additional soil-specific field and laboratory testing, and/or review by an expert panel.

The potential consequences of liquefaction and (if necessary) liquefaction hazard mitigation measures shall be evaluated if the FOS against liquefaction is less than 1.2.

2.2.2 Liquefaction-Induced Movement/Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during and/or following earthquake shaking. Methods to estimate movement/settlement of unsaturated granular deposits are presented in Section 2.8. Liquefaction-induced total ground settlement of saturated granular deposits shall be estimated using at least two of the following methods: Ishihara and Yoshimine (1992), Zhang et al. (2002), Idriss and Boulanger (2008), and Cetin et al. (2009). If a laboratory-based analysis of liquefaction-induced settlement is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the liquefaction-induced vertical settlement in lieu of empirical SPT- or CPT-based criteria. Even when laboratory-based volumetric strain test results are obtained and used for design, the empirical methods shall be used to qualitatively check the reasonableness of the laboratory test results.

It should be noted that all of these estimates are free-field settlements, and structural movement/settlements resulting from soil liquefaction are more important in most of the cases (Bray and Dashti, 2010). Structural movement/settlements may also result from shear-induced movements. Hence, methods that are used for estimating lateral ground movements may be required.

The geotechnical engineer shall compare the estimated movement/settlement values with the allowable deformation values and develop mitigation plans described in Section 2.4, if necessary. The geotechnical engineer shall also consider the long-term, post-construction requirements for earth-and-fill conditions.

2.2.3 Liquefied Residual Strength Parameters

Unless soil-specific laboratory performance tests are conducted as described later in this section, residual strengths of liquefied soil shall be evaluated using at least two of these procedures: Seed and Harder (1990), Idriss and Boulanger (2008), Olson and Stark (2002), and Kramer and Wang (2011). Design liquefied residual shear strengths shall be based on weighted average of the results; Ledezma and Bray (2010) may be used as a reference to select a reasonable weighting scheme.

Results of laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the residual strength in lieu of empirical SPT- or CPT-based criteria. Even when laboratory based test results are obtained and used for design, two of the above empirical methods shall be used to qualitatively check the reasonableness of the laboratory test results. It shall be noted that SPT N fines content corrections for residual strength calculations are different than corrections for liquefaction triggering and settlement.

2.2.4 Surface Manifestations

The assessment of whether surface manifestation of liquefaction (such as sand boils, ground fissures, etc.) will occur during earthquake shaking at a level-ground site that is not within a few hundred feet of a free face shall be made using the method outlined by Ishihara (1985) and shall be compared against results by the method presented in Youd and Garris (1995). It is emphasized that settlement may occur, even with the absence of surface manifestation. The Ishihara (1985) method is based on the thickness of the potentially liquefiable layer (H2) and the thickness of the non-liquefiable crust (H1) at a given site. In the case of a site with stratified soils containing both potentially liquefiable and non-liquefiable soils, the thickness of a potentially liquefiable layer (H2) shall be estimated using the method proposed by Ishihara (1985) and Martin et al. (1991). If the site contains potential for surface manifestation, then use of mitigation methods shall be evaluated.

2.3 Evaluation of Lateral Spreading and Consequences

Lateral spreading shall be evaluated for a site if liquefaction is expected to trigger within 50 feet of the ground surface, and either a ground surface slope gradient of 0.1% or more exists, or a free face conditions (such as an adjacent river bank) exists. Use Shamoto et al. (1998) as a method to assess the maximum distance from the free face where lateral spreading displacements could occur. Historic and paleoseismic evidence of lateral spreading is valuable information that shall also be reviewed and addressed. Such evidence may include sand boils, soil shear zones, and topographic geometry indicating a spread has occurred in the past.

2.3.1 Methodologies for Predicting Lateral Spreading

If there is a free face condition, the post-liquefaction flow failure FOS of an earth slope or sloping ground shall be estimated per Section 2.9.1 below before estimating liquefaction-induced lateral movements. If the post-liquefaction stability FOS is less than 1.0 then empirical or analytical methods cannot generally be used to reliably predict the amount of ground movement.

In order to predict the permanent deformations resulting from the occurrence of lateral spreading during earthquake loading, several methods of analyses are available. These methods of analyses can be categorized into two general types: Empirical Methods and Analytical Methods.

Empirical Methods – The most common empirical methods to estimate lateral displacements are Youd et al. (2002), Bardet et al. (1999), Zhang et al. (2004), Faris et al. (2006) and Idriss and Boulanger (2008). Analysts shall be aware of the applicability and limitations of each method. Lateral displacements shall be evaluated using the Zhang et al. (2004) method and at least one of the other methods described above.

Empirical methods shall be used as the primary means to estimate deformations due to lateral spreading. Multiple models shall be considered, and the range of results shall be reported.

Analytical Methods – For cases where slope geometry, structural reinforcement, or other site-specific features are not compatible with the assumptions of the empirical methods, the Newmark sliding block analyses shall be used. Newmark analyses shall be conducted similar to that described in the seismic slope stability section, except that estimation of the yield acceleration (ky) shall consider strength degradation due to liquefaction. In addition, numerical methods using finite elements and/or finite difference approach may be used.

The geotechnical engineer shall compare the estimated lateral spread values with the allowable deformation values and develop mitigation plans described in Section 2.4, if necessary. The geotechnical engineer shall consider the long-term, post-construction performance requirements for earth-and-fill conditions.

2.4 Analysis for Design of Liquefaction Mitigation Methods

During the liquefaction evaluation, the engineer shall evaluate the extent of liquefaction and potential consequences such as bearing failure, slope stability, and/or vertical and/or horizontal deformations. Similarly, the engineer shall evaluate the liquefaction hazard in terms of depth and lateral extent affecting the structure in question. The lateral extent affecting the structure will depend on whether there is potential for large lateral spreads toward or away from the structure and the influence of liquefied ground surrounding mitigated soils within the perimeter of the structure.

Large lateral spread or flow failure hazards may be mitigated by the implementation of containment structures, removal or treatment of liquefiable soils, modification of site geometry, structural resistance, or drainage to lower the groundwater table.

Where liquefiable clean sands are present, geotechnical evaluations for design shall consider an area of softening due to seepage flow occurring laterally beyond the limit of improved ground a distance of two-thirds of the liquefiable layer thickness, as described in studies by Iai et al. (1988). To calculate the liquefiable thickness, similar criteria shall be used as that employed to evaluate the issue of surface manifestation by the Ishihara (1985) method. For level ground conditions where lateral spread is not a concern or the site is not a water front, this buffer zone shall not be less than 15 feet and it is likely not to exceed 35 feet when the depth of liquefaction is considered as 50 feet, and the entire soil profile consists of liquefiable sand.

The performance criteria for liquefaction mitigation, established during the initial investigation, shall be in the form of a minimum and average penetration-resistance value associated with a soil type (fines content, clay fraction, USCS classification, CPT soil behavior type index Ic, normalized CPT friction ratio), or a tolerable liquefaction settlement as calculated by procedures discussed earlier. The choice of mitigation methods will depend on the extent of liquefaction and the related consequences. In general, options for mitigations are divided into two categories: ground improvement options and structural options.

2.5 Ground Improvement Options

There are many different methods of ground improvement. The five primary methods of ground improvement (and some examples of each of them) to be considered for soil liquefaction mitigation are:

1. Replacement
   - Excavate and replace with compacted fill
2. Vibratory Densification
   - Vibro-compaction
   - Vibro-replacement stone columns (combination of vibration and displacement)
   - Deep dynamic compaction
3. Displacement Densification / Reinforcement
   - Compaction grouting
   - Displacement piles

     Vibro-replacement stone columns (combination of vibration and displacement)
4. Mixing / Solidification
   - Permeation grouting
   - Deep soil mixing
   - Jet grouting
5. Drainage
   - Passive or active dewatering systems
   - Earthquake drains are not permitted for use

The implementation of these techniques shall be designed to fully, or partially, eliminate the liquefaction potential, depending on the requirements of the engineered facility under consideration. Further details, applicability, and limitations of these techniques can be found in Martin and Lew (1999).

2.6 Structural Options

Structural mitigation involves designing the structure to withstand the forces and displacements that result from liquefaction. In some cases, structural mitigation for liquefaction effects may be more economical than soil improvement mitigation methods. However, structural mitigation may have little or no effect on the soil itself and may not reduce the potential for liquefaction. With structural mitigation, liquefaction and related ground deformations will still occur. The structural mitigation shall be designed to produce acceptable structural performance (consistent with the requirements for the two design earthquakes) in terms of liquefaction/lateral spread-induced displacements and structural damage. The appropriate means of structural mitigation may depend on the magnitude and type of liquefaction-induced soil deformation or load.

Depending on the type of structure and amount and extent of liquefaction, common structural options to be considered are as follows:

- Piles or caissons extending to non-liquefiable soil or bedrock below the potentially liquefiable soils
- Post-tensioned slab foundation (appropriate only for small, lightly loaded structures)
- Continuous spread footings having isolated footings interconnected with grade beams
- Mat foundation (appropriate only for small, lightly loaded structures)

Details, applicability, and limitations of these techniques can be found in Martin and Lew (1999). Additional requirements for design of piles in liquefied soil are presented in Section 2.7.

2.7 Seismic Considerations for Lateral Design of Piles in Liquefiable Soils

Seismic considerations for lateral design of pile/shaft design in soils include the effects of liquefaction on the lateral response of piles/shafts and designing for the additional loads due to lateral spread and/or slope failures. Effects of liquefiable soils shall be included in the lateral analysis of piles/shafts by using appropriate p-y curves to represent liquefiable soils. Liquefied soil p-y curves shall be estimated using the static API sand model reduced by a p-multiplier using the method of Brandenberg, et al. (2007) and Boulanger, et al. (2007).

The displacement-based approach for evaluating the impact of liquefaction-induced lateral spreading loads on deep foundation systems that shall follow Caltrans' "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading," dated February 2011 shall be used. However, the liquefaction susceptibility and triggering analyses performed as part of this procedure shall be based on Section 2.1 and Section 2.2, respectively. Similarly, the lateral spread estimates shall be based on Section 2.3. The geotechnical engineer shall compare the estimated lateral spread values with the allowable deformation values and develop mitigation plans described in Section 2.4, if necessary. The geotechnical engineer shall also consider the long-term, post-construction performance requirements for earth-and-fill conditions.

Numerical methods incorporating finite element and/or finite difference techniques may be used to assess pile response in laterally spreading soils.

2.8 Seismic Settlement of Unsaturated Soils

Seismically induced settlement of unsaturated granular soils (dry sands) shall be estimated using procedures provided by Tokimatsu and Seed (1987). Estimated values in terms of total and differential settlements shall be reported.

The geotechnical engineer shall compare the estimated settlement values with the allowable deformation values and develop mitigation plans described in Section 2.4, if necessary. The geotechnical engineer shall also consider the long-term, post-construction performance requirements for earth-and-fill conditions.

2.9 Seismic Slope Stability and Deformation Analyses

Instability of slopes during seismic loading could be due to liquefaction or due to inertial loading, or a combination of both. In this section, instability of both the natural existing slopes and embankment slopes is addressed.

The geotechnical engineer shall compare the estimated deformation values with the allowable deformation values and develop mitigation plans described in Section 2.4, if necessary. The geotechnical engineer shall also consider the long-term, post-construction performance requirements for earth-and-fill conditions.

2.9.1 Liquefaction-Induced Flow Failure

Liquefaction leading to catastrophic flow failures driven by static shearing stresses that result in large deformation or flow shall also be addressed by geotechnical engineers. These flow failures may occur near the end of strong shaking or shortly after shaking and shall be evaluated using conventional limit equilibrium static slope stability analyses. The analysis shall use residual undrained shear strength parameters for the liquefied soil assuming seismic coefficient to be zero (i.e., performed with K_h and K_v equal to zero). The residual strength parameters estimated using the method presented in Section 2.2.3 shall be used. In addition, strength reduction due to cyclic degradation versus strength increase due to the effects of rate of loading shall be considered for normally consolidated clayey layers and non-liquefiable sandy layers. Chen et al. (2006) have discussed the effects of different factors on the dynamic strength of soils. The analysis shall look for both circular and wedge failure surfaces. If the limit equilibrium FOS is less than 1.1, flow failure shall be considered likely. Liquefaction flow failure deformation is usually too large to be acceptable for design of structures, and some form of mitigation will likely be needed. However, structural mitigation may be acceptable if the liquefied material and any overlying crust flow past the structure and the structure and its foundation system can resist the imposed loads.

If the FOS for this decoupled analysis is greater than 1.1 for liquefied conditions, ky shall be estimated using pseudo-static slope stability analysis. The same strength parameters as used during the flow failure analysis shall be used. A new critical failure plane shall be searched assuming both circular and non-circular failure surfaces. Yield acceleration is defined as the minimum horizontal acceleration in a pseudo-static analysis for which FOS is 1.0. Using the estimated ky values, deformations shall be estimated using simplified methods such as Makdisi and Seed (1978) and Bray and Travasarou (2007). Other methods such as Newmark time history method or more advanced methods involving numerical analysis may be used, but shall be checked against the simplified methods.

For pseudo-static analyses to estimate ky values, residual strengths for the liquefied layers and reduced strengths for normally consolidated clayey and saturated sandy layers with excess pore water pressure generation (as described earlier) shall be used. This is generally a conservative approach but is appropriate for initial engineering design. For final design more advanced methods involving numerical analyses may be used to better characterize the initiation of liquefaction and pore pressure generation and subsequent reduction in strength.

2.9.2 Slope Instability Due to Inertial Effects

Pseudo-static slope stability analyses shall be used to evaluate the seismic stability of slopes and embankments due to inertial effects. The pseudo-static analysis consists of conventional limit equilibrium slope stability analysis with horizontal seismic coefficient (K_h) that acts upon the critical failure mass. A horizontal seismic coefficient (K_h) estimated using Bray and Travasarou (2009) and a vertical seismic coefficient, K_v, equal to zero shall be used for the evaluation of seismic slope stability. The Bray and Travasarou (2009) method requires an estimate of allowable deformation to compute K_h. Therefore, for the MCE, an allowable deformation of 6 inches may be used, and for the OBE, the allowable deformation shall be used. For these conditions, the minimum required FOS is 1.0. Alternately, pseudo-static analyses may be performed to estimate K_y values. There is a debate in literature whether the slope failure plane during the pseudo-static analysis shall be fixed based on the results of static analyses or a new failure plane is searched. A new failure plane shall be searched for the pseudo-static analysis. The analysis shall look for both circular and non-circular failure surfaces.

2.9.3 Seismic Slope Deformations

Deformation analyses shall be performed where an estimate of the magnitude of seismically induced slope deformation is required, and the pseudo-static slope stability FOS is less than 1.0. Acceptable methods of estimating the magnitude of seismically induced slope deformation include Newmark sliding block (time history) analysis, simplified displacement charts and equations based on Newmark-type analyses Makdisi and Seed (1978), Bray and Travasarou (2007), and Rathje and Saygili (2008), or dynamic stress-deformation models. These methods shall not be employed to estimate displacements if the post-earthquake static slope stability FOS using residual strengths is less than 1.0, since the slope will be unstable against static gravity loading and large displacements would be expected.

2.10 Downdrag Loading (Dragload) on Structures Due to Seismic Settlement

Downdrag loads on foundations shall be evaluated in accordance with Article 3.11.8 of the AASHTO LRFD Bridge Design Specifications and as specified herein. The AASHTO LRFD Bridge Design Specifications, Article 3.11.8, recommends the use of the non-liquefied skin friction in the non-liquefied layers above and between the liquefied zone(s), and a skin friction value as low as the residual strength within the soil layers that do liquefy, to calculate downdrag loads for the extreme event limit state.

3.0 Fault Rupture / Displacement Design and Evaluation

3.1 General

Evaluation of fault rupture shall be provided for the 2 performance criteria consistent with seismic design criteria set forth in the *Seismic* chapter of the Design Criteria for all faults that meet the capable fault definition as defined in Section 3.1.2. Displacement analyses shall provide designers with location, displacement magnitude, movement direction, and orientation and shall include a description of data uncertainty for consideration within the design process.

Guidelines for analysis are provided below and shall be implemented for design. The design shall implement fault displacement analysis methods for the Maximum Considered Earthquake (MCE) and Operability Performance Earthquake (OBE) events in sufficient detail and reliability so that the design for any required mitigations can be developed. The displacement methods are summarized in Section 3.3.

Additionally, local factors such as near-field effects and topographic amplification shall be considered in estimating ground motions. These values shall be considered in assessing the required mitigation measures to meet the performance criteria.

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These guidelines do not apply to buildings and facilities that do not carry high-speed train loadings. Buildings are subject to Alquist-Priolo requirements which state that buildings cannot be designed and built over Holocene faults.

3.1.1 Qualifications for Capable Fault Rupture Investigation

Geological investigations involving capable fault trace and displacement determination shall be under the direct supervision of a current California licensed Engineering Geologist (CEG).

3.1.2 Capable Fault Definition

Faults subject to these criteria and guidelines are referred to as “capable faults”. Capable faults are defined as a mapped or otherwise known Quaternary fault with evidence of Holocene displacement, structural relationship to related Holocene faults, and/or where data is not sufficient to rule out the presence of Holocene movement.

Where the design of buildings is involved, the CBC definition of Active Faults shall be used and will be subject to all requirements of the Alquist-Priolo.

3.1.3 Seismic Performance Criteria

Fault rupture analysis shall be performed consistent with the Seismic Performance Criteria established in the *Seismic* chapter of the Design Criteria.

3.1.4 Fault Displacement Analysis Methods

This section provides guidance on the methodologies which shall be used to develop surface fault displacements for the two performance criteria. The guidelines address the methodologies to be used for design of HST structures.

3.2 Fault Hazard Zone

The definition of the Fault Hazard Zone (FHZ) is defined as the overall zone within which deformations related to fault rupture may occur and should be considered in the design. This FHZ consists of three components; The primary zone of faulting, a surrounding zone within which secondary or sympathetic displacement has and/or may occur, and the safety zone which is a buffer zone surround the primary and secondary zones that represents the uncertainty of deformations in the future. The information from compiled literature, remote sensing, and field investigations (as required) shall be used to estimate the zone of potential primary rupture. All reasonable mapped fault locations shall be considered as part of the primary zone of fault rupture. The secondary rupture zone shall take into consideration sympathetic or secondary and typically lower displacements. The width of this zone shall encompass paleoseismic trench observations of secondary movement as well as empirical information for similar fault zones and their breadth of secondary movement. The safety zone breadth shall be left to the design team's discretion but will be demonstrated by the designer to be adequate to bracket the uncertainty of future movement(s).

The width of the distributive faulting shall also be assessed for the capable fault in question. That is, the nature of faulting within the overall capable fault zone shall differentiate between the potential for discrete faulting anywhere within the zone as opposed to the distribution of the displacement throughout this zone. A credible explanation will be needed for this differentiation and in the absence of this substantiation, both shall be considered possible and considered within the design until additional data can be obtained to provide the necessary substantiation. The defined fault zone shall conservatively capture potential for future distributive faulting. In addition, the zone containing all mapped faults shall be used to evaluate this spatial variability and thus the overall breadth of this zone and the greater of the two zone widths shall be used for design purposes.

3.3 Fault Displacement Methodology

Fault rupture analysis and design shall be consistent with the *Seismic* chapter of the Design Criteria. For design, the fault displacement values for MCE and OBE events shall be determined and evaluated.

Prior to evaluation of displacement magnitude, the probability of rupture shall be assessed to further define the fault as capable. Contrary to Alquist-Priolo regulations for buildings, the HST system will not necessarily prohibit the construction of non-building facilities at or near known active faults. Buildings will remain subject to California Building Codes (CBC) and thus A-P requirements apply and preclude construction over a Holocene Fault. The probability of rupture shall be evaluated using the seismic performance criteria set forth in the *Seismic* chapter of the Design Criteria. The probability of rupture shall be evaluated for all faults meeting the capable fault definition above. The probability of rupture shall be based on rupture frequency data (where available and reliable)

In general, capable faults that have higher slip rates and/or high frequency return periods will remain classified as capable. If a fault can be effectively demonstrated to have a sufficiently long Return Interval (RI), it may be declassified as capable and may not be subject to the evaluation and mitigation requirements herein. The RI shall be defined as the characteristic (average) return period of the fault and will be compared to the most recent large earthquake. If the return interval (RI) for the fault is approximately equal to or less than the time since the most recent event (RE) and is less than the seismic performance criteria return period (SPC) and these are reliable values, the fault will be remain classified as capable of rupture. This comparison of Return Interval to the most recent event and SPC criteria is expressed in the simple equation as:

If RI – RE < SPC, then rupture is probable and the magnitude of displacement must be evaluated.

If RI - RE ≥ SPC, then rupture is not probable in relation to the seismic performance criteria

Where:

RI = fault return interval

RE = time since the most recent event

SPC = Seismic Performance Criteria Return Period

As an example, if a mapped Quaternary fault is not mapped as Holocene but is on strike with a potentially structurally related fault with evidence of Holocene movement, it shall be classified as capable. If reliable existing or acquired fault characteristic data is available to effectively demonstrate that this fault has a well-constrained RI value of 2,000 years and the most recent event (RE) was 1,500 years ago, the projected future event would be 500 years. Since this value exceeds both the OBE (100 year), but less than the MCE (950 year) return periods, the system needs to be evaluated and mitigated for the NCL (No Collapse Level) performance criteria. It is critical that these fault characteristics be identified as early as possible in the design.

3.3.1 Fault Displacement Magnitude

The fault displacement shall be assessed based on the best available data for design. The displacement value for the MCE (950 year return period) and the OBE (100 year return period) events shall be estimated unless the RI-RE value is greater than the SPC. The displacement magnitude shall be based on the earthquake magnitude (M_w) derived using the Interim Ground Motion (IGM) Analysis methodology, thus assuring consistency between the ground motion value and the ground rupture displacement value for the same fault. Since the IGM methodology appropriately includes the affects of other nearby faults including a background event, the M_w for the fault shall be deaggregated to be representative of movement for only the subject capable fault.

The displacement value shall be computed using the empirical magnitude-displacement correlation developed by Wells and Coppersmith (1994). An alternative correlation can be used if it can be substantiated as being more applicable for the fault characteristics for the evaluated fault. The Youngs et al. (2003) probabilistic fault displacement model shall then be used to independently assess the magnitude of fault displacement (principal and distributive). These values will be compared to the displacement estimated using the Wells and Coppersmith (1994) values. The larger of the two values will be used in the design unless an effective argument can be provided which demonstrates that a certain method is more reliable for the evaluated fault.

Where the subject fault is a “creeping” fault with a high frequency of ruptures, the design will need to accommodate the total displacement during the life expectancy of the HST system by assuring that adequate right-of-way exists and that the cumulative strain can meet or exceed the performance criteria. The displacement analysis shall provide the frequency of displacements, displacement for each event, and the expected cumulative displacement.

3.3.2 Orientation and Direction of Displacement

The orientation of the fault is defined as the alignment and inclination of the fault plane. The direction of displacement is defined as the direction of slip along that plane represented by a vector along the planar surface. The orientation shall be presented as a fault strike value relative to north, and shall be described in degrees of rotation relative to the HST alignment at that location, where applicable. The fault orientation value shall be nearly perpendicular (90°±30°) to HST alignment in order to reduce fault zone length beneath the HST footprint.

The displacement direction for dip-slip faults shall be characterized as being either normal or reverse. Strike-slip faults shall be identified as being either left-lateral or right-lateral. For oblique-slip faults, the displacement of both dip-slip and strike-slip components shall be quantified.

The orientation and direction of displacement of potential ruptures shall be based on all available geologic evidence of fault behavior in the past. If multiple orientations are possible, each shall be considered in design until additional data can be obtained to better constrain this finding. Similarly, the direction of displacement shall be based on geologic data available and any uncertainties or contradictions in data shall be considered in the design until additional data can better define the displacement direction.

3.4 Fault Displacement Design Strategies

3.4.1 General

The displacement obtained from the procedures above shall be used to evaluate the performance of the structures in meeting the Seismic Performance Criteria as defined in the *Seismic* chapter of the Design Criteria.

3.4.2 Analysis Requirements

Structures at or near fault hazards are defined as complex structures. For design, complex structures require either non-linear time history analysis or linear response spectra analysis, based upon the importance classification.

For non-linear time history analysis, the dynamic motions and permanent displacements are to be quantified in separate hazard assessments then combined into a single time history for design.

For linear response spectra analysis, the dynamic spectral response of the structure may be determined separately without consideration of fault displacement. The fault displacement response is then determined statically and added to the dynamic response by superposition.

3.5 Mitigation Classification

Once analyses have been made for structures subject to fault rupture, the systems shall be classified by the mitigation measures required to achieve acceptable performance.

System classification highlights the potential impact to project alignment, design and operation.

- Class A systems – can tolerate expected fault displacements using either standard or special mitigation design in order to meet Seismic Performance Criteria.
- Class B systems – require special mitigation design, but cannot meet standard Seismic Performance Criteria, thus a variance to the minimum criteria and operation is required.
- Class C systems – cannot meet Seismic Performance Criteria and cannot be feasibly mitigated with a variance. Thus, elevated and underground structures may not be used. Such Class C systems shall be composed of at-grade ballasted track with no exceptions.

3.5.1 Variances to Standard Criteria

Damage of systems near or at fault hazard zones is a substantial risk to the HST system. If large fault offsets occur, unavoidable track or structural damage may occur, increasing the risk of train derailment. Where the alignment crosses active faults, system seismic performance criteria may be impractical due to expected large offset displacements each side of the fault."

Thus, for systems with Class B mitigation classification, variances to standard CHSTP performance and operational criteria will be required. Such variances must be specified in writing, and are subject to approval by the Authority.

Examples of performance criteria variances for Class B systems include:

- Exceedence of allowable strain limits for structural components (i.e., variance to the *Seismic* chapter of the Design Criteria)
- Exceedence of allowable deformation limits for the track and structure or exceedence of allowable rail stresses, under an OBE event (i.e.: variance to High-Speed Train and Track Structure Compatibility)

Examples of operational criteria variances for Class B systems include:

- Reduced train speeds near the fault crossing

- Reduced train service near the fault crossing
- Temporary closure for repairs following an OBE event
- Extended closures for repairs following a MCE event

For each Class B mitigation scenario, it is the responsibility of the designer to determine whether variances to standard design criteria are needed, and, if required, submit a Variance Request for approval.

3.5.2 Typical Design Process for Capable Fault Zone Structures

Typical design for elevated or underground structures at fault hazard zones shall consist of:

- Evaluation of site conditions: fault classification and characterization for the two design earthquakes
- Determination of near fault dynamic ground motions, and permanent (i.e.: fault offset) displacements
- Preliminary design based upon the near fault dynamic ground motions and permanent (i.e., fault offset) motions, in order to determine structural demands, and necessary expansion joint displacement and rotational demands
- Identification of fault hazards
- Determination of expected fault displacement demands for OBE and MCE events
- Design non-linear time history analysis or linear response spectra analysis, based upon Structural Classification as defined in the *Seismic* chapter of the Design Criteria
- Development of a bridge or tunnel hazard mitigation plan
- Development of a bridge or tunnel health monitoring system
- Final Mitigation Classification for the system
- Approved Mitigation Class B variances
- At-grade alignments for Class C systems
- Documentation of the design and mitigations.

3.6 Primary Mitigating Strategy at Capable Fault Zones

At fault hazard zones, the primary mitigating strategy is to place the alignment at-grade and oriented as near to perpendicular (90◎±30˚) as feasible to the fault trace, in order to minimize the fault zone length beneath the HST footprint, and allow timely inspections and repairs after an earthquake event.

Elevated and underground construction at fault hazard zones shall, to all practical extents, be avoided.

In order to place the track at-grade, structural improvements such as embankments and retaining walls such as flexible MSE walls may be necessary. Where embankments and retaining walls are needed, consideration shall be made for an increased width of right-of-way. This is in recognition of anticipated damage to the embankments and retaining walls. The increased width shall provide more separation between the tracks and improvements, and add flexibility for realignment work.

For fault offset induced seismic pressures for retaining walls, and modified stability analyses for embankments, refer to the Geotechnical Data Report.

The primary mitigating strategy for trackside Systems facilities, including traction power, train control, communications, and other significant equipment, buildings, huts, and enclosures, is to locate these facilities outside all fault hazard zones.

3.6.1 Earthquake Early Warning System

An earthquake early warning detection system (EEWDS) shall be developed and used system-wide, including additional sensors at fault hazard zone regions. The detection system shall be integrated with the train control, communications and signals systems, and be capable of triggering an appropriate response for at risk trains to bring them to a safe stop as soon as p-waves are detected.

The EEWDS will not be effective if a train is near or at the fault zone due to the short time lapse between the p-wave and s-wave generation. For trains within a few miles of the fault zone, the EEWDS shall be designed to precipitate the braking of trains to a safe stop before they cross potentially damaged track.

The EEWDS implementation shall be coordinated with maintenance and inspection protocols.

3.7 Secondary Mitigating Strategies for Elevated Structures

Where at-grade tracks are infeasible, such as at congested sites, water crossings, or mountainous terrain, then elevated structures may be unavoidable.

For mitigation to Class B systems, variances to standard HST performance and operational criteria will be required. Variances shall be specified in writing and submitted for approval by the Authority.

Realizing the potential for fault rupture damage, mitigating designs which allow rapid track realignment and structural repair shall be pursued. Some secondary mitigating strategies for elevated structures at fault hazard zones follow.

3.7.1 Simple Spans and Elongated Bearing Seats

In order to cost effectively meet train performance requirements, relatively short, simple span structures shall be used. Since such structures, when subject to large fault displacements, are at risk of girder unseating and potential collapse, large and elongated bearing seats shall be used to accommodate the necessary rotations and displacements without introducing significant damaging forces to the piers or girders.

Elongated bearing seats not only provide increased displacement capacity, but also allow for possible post-earthquake realignment capability, thus avoiding costly and time-consuming demolition and reconstruction.

Note that temporary closure, track realignment, and repair reconstruction may be unavoidable, even for the most effective designs.

3.7.2 Seismic Isolation and Dissipation Devices

For longer and continuous span bridges at fault hazard zones, seismic isolation and response modification systems may be considered. Isolation systems such as friction pendulum bearings, capable of resisting both the dynamic and permanent offset displacements, have been successfully used on long viaducts. Other isolation systems may be equally viable.

Due to the stringent high-speed train serviceability requirements, careful attention must be made when using isolation and response modification systems, especially when considering their response to normal service loads.

3.7.3 Large Diameter Monopile Foundations

Where the fault zone is well defined, and the design has confirmed that fault rupture will not rupture through the piers, traditional multi pile caps may be used.

Where the fault zone is not well defined, or is known to exist over a wide area, then large diameter monopile foundations shall be considered. The use of this type system will minimize the risk of damage due to a fault rupture passing directly through a traditional multi pile cap.

3.7.4 Self Centering Columns

For near fault regions, where dynamic motions may be very intense, the use of self-centering columns founded upon a traditional multi pile cap may be considered. Self-centering columns have been shown to be capable of reducing post-earthquake residual displacements.

Self-centering columns are concrete columns with vertical, concentric unbonded post-tensioned tendons. Research has shown that the tendons effectively apply a restoring force, thus limiting residual post-earthquake displacements. The use of unbonded vertical reinforcement, and steel jackets at the plastic hinge zones, further add to self-centering column performance.

3.8 Secondary Mitigating Strategies for Underground Structures

Where at-grade tracks are infeasible, such as at congested sites, water crossings, or mountainous terrain, underground structures may be unavoidable.

For mitigation Class B systems, variances to HST performance and operational criteria will be required. Variances shall be specified in writing and submitted for approval by the Authority.

Secondary mitigating designs for underground structures which allow rapid track realignment and structural repair shall be pursued. Some secondary mitigating strategies for underground structures at fault hazard zones follow.

3.8.1 Fault Chambers

Where tunnels cross known faults with large offset displacements, local use of a larger tunnel cross section shall be considered. The larger cross section shall be sized based upon the predicted direction and magnitude of offset in order to allow clear passage and realignment of track post-earthquake.

It may be necessary to extend the length of the larger cross section beyond the fault zone length for track realignment purposes.

3.8.2 Increased Width at Trenches

Where trenches exist at known fault crossings, consideration shall be made for increased width in recognition of anticipated damage to the walls. The increased width will provide more separation between the tracks and damaged walls, allow room for construction access, and provide additional flexibility for realignment work.

3.8.3 Tunnel Lining System at Lesser Faults

Where tunnels cross known lesser faults with smaller offset displacements, a tunnel lining system shall be considered which allows rapid repair. Shotcrete and dowel rock reinforcement systems have been used previously for this situation. If lining damage occurs, then additional dowels and shotcrete can be installed post-earthquake to allow service resumption.

3.9 Other Primary Structures

3.9.1 Ductbank Fault Chambers

Where ductbanks cross known faults with large offset displacements, the use of an oversized buried containment structure to house the ductbank shall be considered. The size of the containment structure shall be based upon the predicted direction and magnitude of offset in order to maintain service.

It may be necessary to extend the length of the ductbank containment structure beyond the fault zone to maintain serviceability.

3.9.2 Service Loops

Service loops or extra lengths of fiber optic or other communication lines in ductbanks shall be provided within fault zones.

3.10 Hazard Mitigation Plan

When design solutions to minimize risk levels at fault hazard zones are not possible, mitigation measures shall be developed in accordance with the Hazard Management and Resolution Process prescribed by the project-wide System Safety Program Plan (SSPP) and may include the following:

- Definition of expected structural damage
- Health monitoring system
- Earthquake Early Warning Detection system
- Emergency access and evacuation plan
- Inspection Protocol
- Methods of repair
- Estimated down time
- Alternative routes, if any.

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